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Abstract

The objective of this paper is to document progress on new cold-formed steel provisions for the forthcoming edition of ASCE 41, Seismic Evaluation and Retrofit of Existing Buildings. The current edition of ASCE 41 (2013) is weak with respect to the application of cold-formed steel and provides only limited information on cold-formed steel framed buildings, shear walls, members, and connections. The emphasis in this paper is on cold-formed steel framed shear walls, and the development of modeling parameters that characterize the backbone shear-deformation response, and acceptance criteria that provide allowable demand-to-capacity ratios (*m*-factors) for the shear walls based on a broad evaluation of existing data. Significant additional work has been developed to update ASCE 41; including, developing descriptions of benchmark buildings framed from cold-formed steel and how damage and deterioration is observed in these buildings. These descriptions are necessary in the evaluation process and exist for other building materials in ASCE 41 (2013), but not for cold-formed steel framing. In addition, modeling parameters and acceptance criteria are provided for individual cold-formed steel members in flexure and steel-to-steel connections. The paper provides a description of the collected experimental data and the procedures employed for developing modeling parameters and acceptance criteria, and provides the developed factors in summary form as currently being finalized through the ASCE 41 balloting process. The long-term goal of this effort is to further enable performance-based seismic design for buildings framed from cold-formed steel.

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Introduction

Most building structural engineers are aware of the seismic design provisions in ASCE 7 (2010). For cold-formed steel (CFS) framed buildings the equivalent lateral force (ELF) procedure of ASCE 7 is most commonly used. The ELF method in ASCE 7 requires an estimation of building mass and period that once suitably modified by seismic response modification coefficients (e.g. R) results in an estimate of the demand base shear and its distribution along the height of the building. The lateral force resisting system must be designed against these demands, and consideration is also given to overstrength and deflection in the design process. Alternative procedures using nonlinear static pushover analysis, linear dynamic analysis, and nonlinear dynamic (time history) analysis, are all allowed, but are uncommon for CFS framing due to difficulties including a lack of required information for completing the modeling accurately.

ASCE 41: Seismic Evaluation and Retrofit of Existing Buildings (2013), provides an alternative seismic design procedure that despite its name can be used both for existing or new design. ASCE 41 is a performance-based standard and provides differing solutions based on the designers objective for their building (or retrofit): immediate occupancy, life safety, or collapse prevention. Once the performance level is set the demand is determined (e.g. for an immediate occupancy level a particular base shear and distribution for use in a linear static procedure is set) and the building components are evaluated for that demand. Each component is characterized as either deformation-controlled or force-controlled and appropriate demand-to-capacity ratios are compared against allowable demand-to-capacity ratios known as m-factors. Common m-factors are near 3, but vary considerably. Note, m-factors are specific to linear static analysis, similar demand-to-capacity ratios for deformation and force are provided for other analysis procedures.

The ASCE 41 m-factors are similar in spirit to the ductility-based portion of the R factor used in ASCE 7, but direct comparisons are not possible. ASCE 7 (2010) and ASCE 41 (2013) do not result in the same design solutions even for new buildings (Harris and Speicher 2015). The closest comparison that can be made to the intended structural performance objective of ASCE 7 (2010), i.e. collapse prevention against an MCE event, is selection of the collapse prevention objective and the BSE-2 hazard in ASCE 41 (Harris and Speicher 2015). Even still, the results are highly site specific and one finds that despite resting on the same knowledge basis ASCE 7 (2010) and ASCE 41 (2013) result in different designs.

As a standard, ASCE 41 is growing in importance in the United States. For one, ASCE 41 provides a codified method that includes multiple performance objectives. For organizations or owners that seek performance beyond the collapse prevention levels of ASCE 7, the methods of ASCE 41 provide a path. Second, ASCE 41 provides a codified procedure for seismic retrofits. The need for seismic retrofits continues to grow, as does the geographic locations where such retrofits are being considered. Third, and finally, ASCE 41 provides a detailed means to employ nonlinear analysis, specifically nonlinear static pushover analysis, in an organized method to improve upon ELF-based (linear static) designs and better represent actual structural behavior.

During the process of updating ASCE 41 for the 2013 version a stark lack of knowledge in the application of this standard for CFS framing was identified, as well as a need to develop a better solution. Expansion of the scope of ASCE 41 to cover CFS and CFS framing requires (a) existing CFS construction be fully accounted for in a retrofit seismic design, (b) new CFS construction be utilized where appropriate in seismic retrofits, and (c) new seismic design is enabled to use CFS. This paper addresses the work that was completed to help meet these goals in the development of the 2017 version of ASCE 41.

The paper begins with the methodology that ASCE 41 employs to characterize the acceptable performance of structural components. This is followed by a detailed discussion of the development and application of a database on CFS framed shear walls and strap-braced walls to establish acceptance criteria and modeling parameters for these critical CFS systems in ASCE 41's format. Additional information on acceptance criteria for CFS members in flexure and steel-to-steel connections follows the work on shear walls. Finally, a discussion of the development of benchmark buildings and other overall changes for CFS and CFS framing needed in ASCE 41 are provided.

Force-deformation (Q-Δ) and demand-to-capacity (*m*-factor) ratios

A central premise of the structural analysis that underpins ASCE 41 is the ability to define the idealized force-deformation (Q-Δ) response of structural components as illustrated in Figure 1. For example, for CFS framing a key primary component may be a CFS framed shear wall and the force Q would be the lateral shear on the wall and the deformation Δ the related lateral displacement. The Q-Δ response in this example is the backbone of the hysteretic response of the wall. This Q-Δ response is idealized to a set of linear segments, as illustrated in Figure 2. ASCE 41 defines three potential performance levels: immediate occupancy (IO), life safety (LS), or collapse prevention (CP). These performance levels are utilized in the creation of acceptance criteria, aligned with the performance levels, that are defined as a function of key deformation limits as conceptually illustrated in Figure 1. Note, primary (P) and secondary (S) components of the structure employ different acceptance criteria (deformation limits) as illustrated in Figure 1.

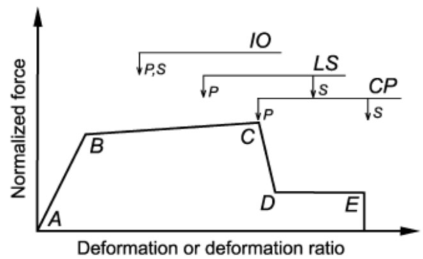

Figure 1. Acceptance Criteria illustration per ASCE 41 Section 7.6.3

The application of the idealized Q-Δ response and the implementation of the acceptance criteria depends on the structural analysis performed. When a nonlinear static (pushover) analysis is used to estimate the demands on the structure the Q-Δ response is utilized directly in the model and ASCE 41 provides the modeling parameters (a – c) as illustrated in Figure 2 along with methods for determining initial stiffness and peak strength to define the full response curve. Deformations in the model are compared against deformation-based acceptance criteria that depend on the performance level.

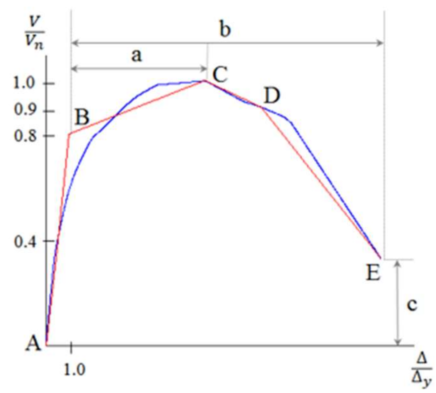

Figure 2. Comparison of shear wall backbone and idealized ASCE 41 response with deformation points $\Delta_A - \Delta_E$ and modeling parameters *a*-*c* illustrated

When linear static analysis is performed for the evaluation then only the elastic stiffness is employed and deformations have to be inferred from the developed force levels. Thus, force-based demand-to-capacity ratios are employed – this implies certain assumptions about the nature of the dynamic displacements, conceptually similar to the "equal displacement rule" that the magnitude of nonlinear deformations in a nonlinear time history analysis are similar to those in a linear dynamic analysis. For linear static analysis the acceptable demand-to-capacity ratios are known as *m*-factors and are defined in terms of the idealized deformation points as given in Table 1. Note, acceptance criteria for nonlinear static analysis are similar, but without the additional 0.75 pre-factor enforced for *m*-factors due to the additional uncertainty inherent in using a linear static procedure to estimate a nonlinear response. ASCE 41 (2013) does not provide the Q-Δ response nor the *m*-factors for CFS or CFS framing. Thus, an essential feature of the proposed updates to ASCE 41 is to gather existing data and develop these response predictions and acceptance criteria.

Table 1. Definition of *m*-factors as acceptance criteria (ASCE 41 Section 7.6.3)

	Primary	Secondary
m_{IO}	$0.75 \times 0.75 \times 0.67 \times \Delta_C/\Delta_B$	$0.75 \times 0.67 \times \Delta_C/\Delta_B$
m_{LS}	$0.75 \times 0.75 \times \Delta_C/\Delta_B$	$0.75 \times 0.75 \times \Delta_E/\Delta_B$
m_{CP}	$0.75 \times min\left(\Delta_C/\Delta_B, 0.75 \times \Delta_E/\Delta_B\right)$	$0.75 \times \Delta_E/\Delta_B$

Experimental database of CFS framed shear walls and strap-braced walls

CFS seismic force-resisting systems are defined in AISI S400 (previously AISI S213) and include CFS framed shear walls with wood structural panel (WSP), steel sheet (SS), gypsum board (GB), or fiberboard (FB) sheathing, and CFS framed strap-braced walls. Note, AISI S400 also includes CFS special-bolted moment frames, not discussed here further. AISI S400 provides nominal shear capacity and in most cases provisions to predict the displacement up to that capacity for these systems. However, AISI S400 does not provide post-peak displacements nor the other specifics of the deformation that are necessary for developing the Q-Δ and resulting *m*-factors that ASCE 41 requires.

The strength of cold-formed steel shear walls and strap-braced walls has been established through testing. The tests were conducted on single story walls connected at their base to a foundation, and loaded with in-plane shear at the top. Tests are generally performed to ASTM standards: ASTM E564 (2006) for monotonic tests and ASTM E2126 (2011) for cyclic tests. ASTM E2126 provides several different cyclic testing protocols and early CFS shear wall testing was conducted to the Sequential Phase Displacement (SPD) protocol while more recent testing (since the late 1990's) have generally been tested to the CUREE loading protocol (Krawinkler et al. 2000). Typical shear wall test setups and response at or near peak displacement are provided in Figure 3.

(a) typical one-sided WSP specimen in test rig (Hikita 2006) | (b) strap-braced wall after testing showing yielding in strap (Comeau 2008) | (c) steel sheet shear wall exhibiting shear buckling (Balh 2010)

Figure 3. Observed response of common cold-formed steel framed shear walls

To develop the proposed ASCE 41 force-deformation response curves and *m*-factors the data underlying the walls in the AISI S400 standard and additional data in the open literature, over 500 tests, were gathered, including: Al-Kharat and Rogers (2005, 2006), Balh and Rogers (2010), Blais (2006), Boudreault (2005), Branston (2004), Chang (2004), Comeau (2008), DaBreo (2012), El-Saloussy (2010), Elhajj (2005), Hikita (2006), Kochkin and Hill (2006), Liu et al. (2012), Lu (2015), Morello (2009), Ong-Tone (2009), Rokas (2006), Serrette et al. (1997), Shamin (2012), Velchev (2008), Yu and Chebn (2009), Uy et al. (2007), and Zhao and Rogers (2002). The nature of the type of wall tested and the number of available monotonic and cyclic tests is summarized in Table 2.

Table 2. Count of available test data distributed across wall types

		Loading Protocol		**Wall Aspect Ratio**			
Sheathing	**Detail**	**Cyclic**	**Monotonic**	**4**	**2**	**1**	**Total**
WSP	CSP	45	52	12	63	22	97
	DFP	13	13		26		26
	OSB	40	37	24	53		77
	Plywood	8			8		8
STRAP	X	41	52	6	17	70	93
	Dogbone	2	6			8	8
	+GYP	8	8		16		16
SS	-	84	93	54	97	5	177[a]
GYP	1 Ply	8	9		17		17
	2 Ply	4	4		8		8
FB	-	8	4	2	2	8	12
Bare	-		1		1		1
	Total	**261**	**279**				**540**

a. not all SS tests at standard aspect ratios, 21 tests at aspect ratio of 1.3

Typical force-deformation response for CFS-framed walls resisting shear are provided in Figure 4. Note, that the backbone response of each test is highlighted in Figure 4 as this data is foundational to the ASCE 41 idealizations. The variety of tested response is large; however, Figure 4 attempts to provide an overview by selecting walls on the lower end of strength capacity (noted as light) and on the stronger end (noted as heavy) and three major wall types: WSP, SS, and strap-braced. Despite differing greatly in their mechanics, all the wall types provided in Figure 4 exhibit strongly pinched cyclic response. However, they exhibit different post-peak response, which will be reflected in the ASCE 41 modeling parameters and acceptance criteria for the different wall types.

Q-Δ and *m*-factors for CFS framed shear walls and strap-braced walls

To develop the idealized ASCE 41 Q-Δ response and *m*-factors first the backbone response of all the shear wall data must be determined. The cyclic data is averaged to provide response in only one direction. Then, the idealized linear segments of ASCE 41 are fit to this data. The fit is determined as shown in Figure 2. The initial linear stiffness is established at 40% of the peak capacity, and this linear stiffness is then extended to 80% of the peak capacity (point B). The second linear segment extends from B to the peak shear and displacement at peak shear (point C). The third linear segment extends to the post-peak displacement at 90% of peak capacity (point D). The final linear segment extends to the end of the stable response (point E).

Once the idealized response curve (Figure 2) is established for each test the *m*-factors and other acceptance criteria can be developed for each test per Table 1. It was determined that the monotonic response gave similar or slightly more conservative average *m*-factors than the cyclic response, e.g. see Table 3, and as a result the monotonic data was kept in the evaluation of the acceptance criteria.

Table 3. Impact of loading protocol on *m*-factors for CFS shear walls with WSP

	Primary component m-factors by performance level								
	IO			**LS**			**CP**		
WSP	Loading protocol			Loading protocol			Loading protocol		
Sheathing	**CUREE**	**Mono**	**SDP**	**CUREE**	**Mono**	**SDP**	**CUREE**	**Mono**	**SDP**
CSP	1.4	1.4	1.4	2.1	2.1	2.1	2.7	2.5	2.7
DFP	1.2	1.2		1.9	1.9		2.5	2.3	
OSB	1.8	1.7	1.2	2.7	2.5	1.7	3.6	3.2	2.0

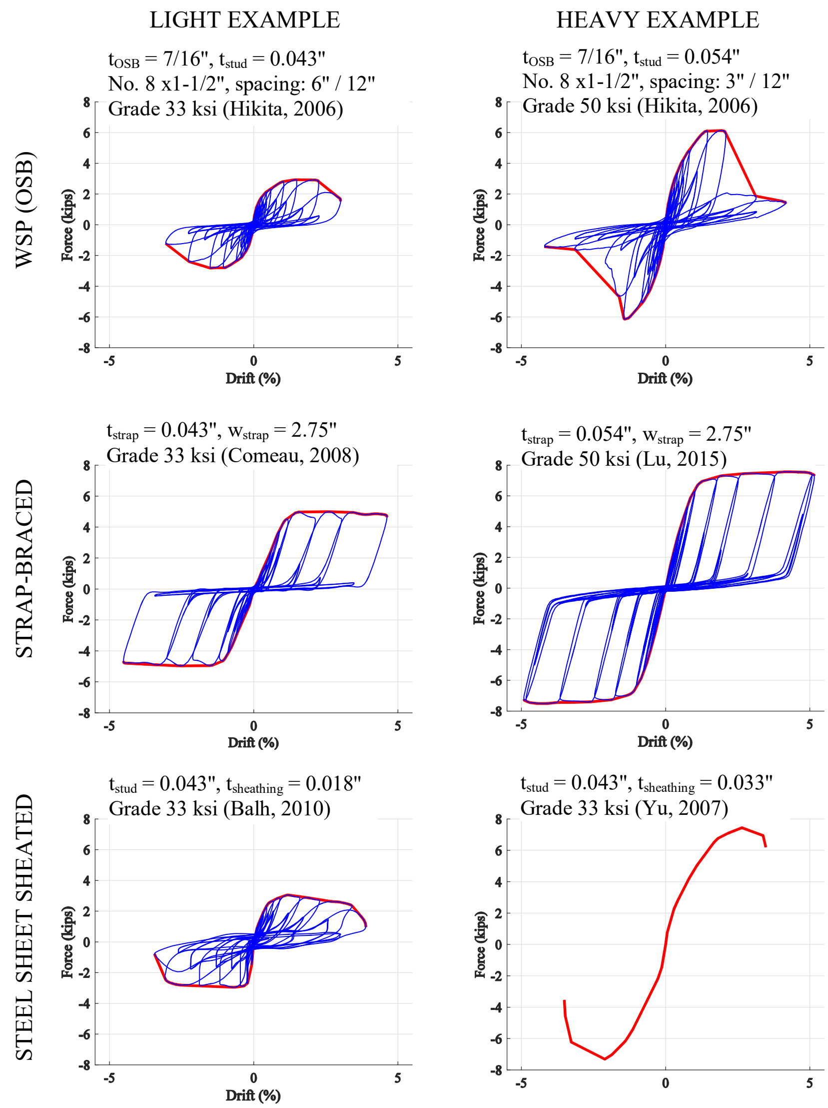

Figure 4. Hysteretic response recorded in typical cyclic shear wall testing for common shear wall types used in cold-formed steel framing. Examples across the tested spectrum provided. (Complete hysteretic response for heavy example steel sheet sheathed shear wall available in Yu (2007), authors have digitized and provided backbone response only).

The majority of the shear wall testing has been conducted on single-sided walls, i.e. where the sheathing or strap was on one side of the wall only. For example, all of the CFS-framed walls with WSP are single-sided. Limited data on double-sided strap-braced walls, steel sheet sheathed walls, and gypsum board sheathed walls all indicate modestly improved *m*-factors (greater post-peak deformation ductility) for double-sided walls over single-sided walls. The proposed ASCE 41 is silent about this fact given the limited information, but the engineer should be aware that doubled-sided walls do appear to have improved performance.

The aspect ratio (wall height / wall width) of tested shear walls is summarized in Table 2. In general, wide walls, aspect ratio of 1, perform better (higher *m*-factors) than narrow walls. As a result, the *m*-factors were separated by aspect ratio where warranted by the data. In some cases, for example CFS-framed shear walls with OSB, performance with the narrow, aspect ratio of 4, walls was modestly better than at an aspect ratio of 2, and the data was left aggregated.

Initial evaluation of the strap-braced walls indicated high variation in the determined *m*-factors. Closer investigation revealed that the Al-Kharat and Rogers (2005) results were the source. In these tests the straps were not capacity-designed and fractured prior to yielding. Subsequently, AISI S213 and today AISI S400 explicitly required capacity protection of the straps and all subsequent testing resulted in the expected performance. Thus, these 16 tests were removed from the 117 tests on strap-braced walls in determining *m*-factors.

The resulting average *m*-factors for linear static analysis, and modeling parameters and acceptance criteria for nonlinear static analysis are provided in Tables 4 and 5 for all shear walls and strap-braced walls. The *m*-factors, modeling parameters, and acceptance criteria are provided to an accuracy of 0.1. This precision overstates the accuracy of the provisions, but is necessary for maintaining the ordinality in the factors across the performance levels. An assessment of variation in the provided *m*-factors indicates that the coefficient of variation (standard deviation/mean) for the *m*-factors for the WSP shear walls at the CP level is between 15 and 30%.

The proposed ASCE 41 provisions include Tables 4 and 5 and guidance on how to develop the linear elastic stiffness and strength. In general, AISI S400 is referenced for determining the nominal stiffness and strength with additional modifications specific to ASCE 41’s reliability basis (e.g., expected strength in ASCE 41 vs. available strength in AISI S400).

Developing m-factors and Q-Δ Provisions for CFS flexural members

For flexural members, the work of Ayhan and Schafer (2016) was employed to provide closed-form solutions to the backbone curve (Fig. 5) and as a result the modeling parameters and acceptance criteria. The provided expressions are unique to ASCE 41 and provide a basic building block for evaluating ductility of cold-formed steel members as individual components.

Table 4. Proposed Numerical Acceptance Factors for Linear Procedures of CFS Light-Frame Components per ASCE 41

		***m*-factors**				
		Primary			**Secondary**	
Component/Action	**Limitation**	**IO**	**LS**	**CP**	**LS**	**CP**
CFS Light-Frame Construction Shear Walls[a,b]	Height/Width Ratio (h/b)					
Structural 1 Plywood	≤ 2	1.2	1.9	2.4	2.8	3.7
Oriented Strand board (OSB)	≤ 4	1.7	2.5	3.3	4.2	5.6
Canadian Soft Plywood (CSP)	≤ 2	1.4	2.1	2.7	3.1	4.1
"	4[c]	1.3	1.9	2.3	2.3	3.1
Douglas Fir Plywood (DFP)	≤ 2	1.2	1.9	2.4	2.8	3.7
Steel Sheet Sheathing	≤ 2	1.5	2.2	2.9	5.2	6.9
"	4[c]	1.1	1.6	1.9	1.9	2.5
Gypsum Board Panel	≤ 2	2.3	3.5	4.6	8.3	11.1
Fiberboard Panel	≤ 2	1.1	1.7	2.3	2.8	3.7
Plaster on metal lath	≤2.0	1.7	3.7	4.4	3.7	5.0
CFS Light-Frame Construction Strap-braced Walls[a,b]	Height/Width Ratio (h/b)					
Flat strap	≤ 2	3.0	4.4	4.9	5.3	7.1
Dogbone strap	≤ 2	3.8	5.7	6.2	6.2	8.3
Flat strap with 1 or 2 plys of Gyp	≤ 2	1.2	1.8	2.4	3.8	5.1
CFS Members						
CFS Member in Flexure		$0.38\frac{\theta_2}{\theta_y}$	$0.56\frac{\theta_2}{\theta_y}$	$0.75\frac{\theta_2}{\theta_y} \leq 0.56\frac{\theta_4}{\theta_y}$	$0.56\frac{\theta_4}{\theta_y}$	$0.75\frac{\theta_4}{\theta_y}$
CFS Member in Compression		[Reserved]				
CFS Connections	fastener					
Screws – steel to steel (33 to 97 mil sheet)[d]	#8, #10, #12	2.5	4.0	4.5	15	20
Screws – wood to steel		[Reserved]				
Bolts – steel to steel		[Reserved]				

[a] Components are permitted to be classified as secondary components or nonstructural components, subject to the limitations of ASCE 41 Section 7.2.3.3. Acceptance criteria need not be considered for walls classified as secondary or nonstructural.
[b] Components with aspect ratios exceeding maximum listed values are not considered effective in resisting seismic forces.
[c] Linear interpolation between aspect ratios for determination of m-factors is permitted.
[d] Median values are provided, variation across sheet thickness and fastener size can be significant.

Table 5. Proposed Numerical Acceptance Factors for Nonlinear Procedures of CFS Light-Frame Components per ASCE 41

		Modeling Parameters			Acceptance Criteria		
		Δ/Δ_y		Residual strength ratio	Δ/Δ_y		
Component/Action	**Limitation**	a	b	c	IO	LS	CP
CFS Light-Frame Construction Shear Walls[a,b]	Height/Width Ratio (h/b)						
Structural 1 Plywood	≤ 2	2.3	4.0	0.3	3.0	3.7	4.0
Oriented Strand board (OSB)	≤ 4	3.4	6.5	0.3	4.2	5.6	6.5
Canadian Soft Plywood (CSP)	≤ 2	2.7	4.5	0.3	3.3	4.1	4.5
"	4c	2.4	3.2	0.6	2.8	3.1	3.2
Douglas Fir Plywood (DFP)	≤ 2	2.3	4.0	0.3	3.0	3.7	4.0
Steel Sheet Sheathing	≤ 2	2.9	8.2	0.6	3.8	6.9	8.2
"	4c	1.8	2.5	0.8	2.3	2.6	2.5
Gypsum Board Panel	≤ 2	5.2	13.8	0.6	6.1	11.1	13.8
Fiberboard Panel	≤ 2	2.0	3.9	0.4	3.0	3.7	3.9
Plaster on metal lath	≤2.0			0.2	1.9	4.4	4.0
CFS Light-Frame Const. Strap Braced Walls[a,b]	Height/Width Ratio (h/b)						
Flat strap	≤ 2	6.9	8.4	0.8	5.9	7.1	8.4
"	4[c]						
Dogbone strap	≤ 2	9.2	10.1	0.6	7.4	8.3	10.1
Flat strap w/ 1 or 2 ply Gyp	≤ 2	2.2	5.8	0.9	3.2	5.1	5.8
CFS Members							
CFS Member in Flexure		$\frac{\theta_2}{\theta_y}-\frac{\theta_1}{\theta_y}$	$\frac{\theta_4}{\theta_y}-\frac{\theta_1}{\theta_y}$	$\frac{M_4}{M_y}$	$\frac{\theta_2}{\theta_y} \le 0.67\frac{\theta_4}{\theta_y}$	$0.75\frac{\theta_4}{\theta_y}$	$\frac{\theta_4}{\theta_y}-\frac{\theta_1}{\theta_y}$
CFS Member in Compression		[Reserved]					
CFS Connections							
Screws – steel to steel (33 to 97 mil sheet)[d]		5	25	0.9	6	20	25
Screws – wood to steel		[Reserved]					
Bolts – steel to steel		[Reserved]					

[a] Components are permitted to be classified as secondary components or nonstructural components, subject to the limitations of Section 7.2.3.3. Acceptance criteria need not be considered for walls classified as secondary or nonstructural.
[b] Components with aspect ratios exceeding maximum listed values are not considered effective in resisting seismic forces.
[c] Linear interpolation between aspect ratios for determination of m-factors is permitted.
[d] Median values are provided, variation across sheet thickness and fastener size can be significant

Developing m-factors and Q-Δ Provisions for CFS steel-to-steel connections

Recent connection testing of Moen et al. (2016) provides data that was utilized to provide basic guidance on steel-to-steel shear connections varying from 0.033 in. (0.84 mm) to 0.097 in. (2.46 mm) thick. Moen et al. (2016) provides Q-Δ response in nearly ready-for-ASCE 41 format. Median modeling parameters and acceptance criteria (*m*-factors) across the different ply thickness and fastener types tested were selected to provide basic guidance. Expected strength may be established from AISI S100 for this connection.

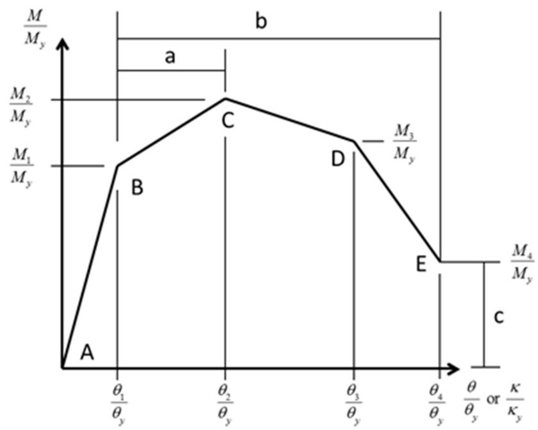

Figure 5. Moment-rotation relation for CFS members in bending

Overall changes to ASCE 41 to enable CFS framing

ASCE 41 utilizes the concept of common building types for assessing damage and deterioration necessary for seismic retrofit studies. This goes beyond ASCE 7's definition of seismic force resisting systems and encompasses the entire building system. ASCE 41 (2013) does not include any common building types with CFS or CFS framing. Therefore, a primary activity in the proposed revisions is the definition of common building types for CFS and CFS framing. Guidance on common building types of CFS light frame construction for residential occupancies, and commercial and industrial occupancies is proposed for addition to ASCE 41 Chapter 3 (2017), and an excerpt is provided in Table 6. The provided descriptions are based largely upon engineering judgment, experience with typical CFS light frame construction built over the past 20 years, and comparisons with similar common building types – wood light frame and structural steel braced frames. Definition of common building types is critical to allow the use of ASCE 41's Tier 1 and Tier 2 seismic retrofit and evaluation procedures. Two sets of buildings types – CFS1 and CFS2 have been defined for two major classes of CFS seismic force resisting systems (SFRS) shear walls and strap braced walls (see Table 6). Additional building types CFS3 and CFS4 are used to reflect the two different CFS SFRS within the commercial and industrial occupancy category. Finally, language is added to the steel braced frames (S2/S2a in ASCE 41), concrete shear walls (C2 in ASCE 41), and reinforced masonry bearing walls (RM1 in ASCE 41) that permits CFS light frame construction to carry gravity loads and to transfer seismic loads to the designated seismic resisting system, as is often the case in actual buildings.

Table 6. Excerpt from proposed addition to ASCE 41 Table 3-1 for CFS

Cold-Formed Steel Light Frame Construction, Residential	
CFS1 (Shear Wall System)	These buildings are single- or multi-family dwellings, one or more stories high. Building loads are light and the framing spans are short. Floor and roof framing consists of cold-formed steel joists or rafters on cold-formed steel studs spaced no more than 24 in. apart. The first-floor framing is supported directly on the foundation system or is raised up on cripple studs and post-and-beam supports. The foundation is permitted to consist of a variety of elements. Chimneys, where present, consist of solid brick masonry, masonry veneer, or cold-formed steel frame with internal metal flues. Seismic forces are resisted by wood structural panel or metal deck diaphragms and wood structural panel sheathed shear walls or steel sheet sheathed shear walls. Floor and roof sheathing consists of wood structural panels. Interior surfaces are sheathed with plaster or gypsum board.
CFS2 (Strap Braced Wall System)	These buildings are single- or multiple-family dwellings one or more stories high. Building loads are light and the framing spans are short. Floor and roof framing consists of cold-formed steel joists or rafters on cold-formed steel studs spaced no more than 24 in. apart. The first-floor framing is supported directly on the foundation system or is raised up on cripple studs and post-and-beam supports. The foundation is permitted to consist of a variety of elements. Chimneys, where present, consist of solid brick masonry, masonry veneer, or cold-formed steel frame with internal metal flues. Seismic forces are resisted by diaphragms with wood structural panels or metal deck and walls with diagonal flat strap bracing. Floor and roof sheathing consists of wood structural panels. Interior surfaces are sheathed with plaster or gypsum board.

Also proposed for ASCE 41 (2017) is that commentary guidance on structural performance levels and illustrative damage descriptions be added to the existing table in Chapter 2 for three of the most common cold-formed steel light frame seismic force-resisting systems – shear walls with WSP, or SS, and strap-braced walls. An excerpt of the proposed addition for CFS framed shear walls with WSP is provided in Table 7. The proposed definitions are based upon engineering judgment and observation of CFS light frame test specimens.

Table 7. Excerpt from Proposed addition to ASCE 41 Table C2-4 for CFS (Provisions are similar to Wood stud walls see ASCE 41)

Seismic-Force-Resisting System	Type	Structural Performance Levels		
		Collapse Prevention (S-5)	**Life Safety (S-3)**	**Immediate Occupancy (S-1)**
Cold-formed steel light frame construction with wood structural panel shear walls	Primary elements	Connections loose. Screw hole deformation at panels and members. Some screws withdrawn. Significant yielding and distortion of members. Significant damage to panels and/or anchors. Loose connections of hold downs to studs.	Moderate loosening of connections and minor yielding of members. Some damage to panels.	Distributed minor hairline cracking of gypsum and plaster veneers applied to shear walls, primarily at door and window openings.
	Secondary elements	Sheathing sheared off. Members yielded with significant distortion. Many broken windows, major sheetrock cracks, inoperable doors.	Connections loose. Screws partially withdrawn. Some yielding of members and damage to panels. Moderate cracking of sheetrock, several broken windows.	Same as for primary elements.
	Drift	Transient drift sufficient to cause extensive nonstructural damage. Significant permanent drift.	Transient drift sufficient to cause nonstructural damage. Noticeable permanent drift.	Transient drift that causes minor or no nonstructural damage. Negligible permanent drift.

Additional guidance on patterns of defects and deterioration, default yield strengths, and benchmark buildings (buildings that, if designed and constructed in accordance with certain recognized standards do not require additional seismic evaluation) is proposed for addition to ASCE 41 Chapter 4. These provisions provide an abbreviated history of the adoption of CFS systems into building codes and standards. The first provisions for CFS SFRS are found in the 1997 UBC, Section 2220, for Seismic Zones 3 and 4. These provisions are for wood structural panel (WSP) shear walls only. While the 1997 NEHRP Provisions (FEMA 302) contained basic requirements for SFRS with CFS shear walls and diagonal strap braced walls, the strap braced wall system did not become a recognized SFRS with its own seismic design parameters until the 2002 edition of ASCE 7 and the 2003 editions of both the NEHRP Provisions (FEMA 450) and the IBC. Consequently, the 2003 editions have been used as the basis for the benchmark building with a strap braced wall system. Light frame construction with shear walls of steel sheet sheathing were first recognized in the 2000 edition of the IBC and the 2002 edition of ASCE 7. Interestingly, the NEHRP Provisions never separately called out SS sheathing from WSP sheathing. Rather, requirements in the NEHRP provisions for CFS were focused on WSP solutions. Therefore, the NEHRP entries in the existing ASCE 41 table have been limited to CFS framing with WSP sheathing. FEMA 356 also focused on shear walls with WSP, therefore the same limitation has been added to those entries. However, the IBC entries are not limited, since both WSP and SS sheathing were recognized options.

It is proposed that in ASCE 41 (2017) Chapter 5, provisions be added providing Tier 2 deficiency-based evaluation procedures that apply to CFS light frame shear walls. Additional provisions are added for CFS strap braced walls. Minor modifications are also proposed to recognize CFS light frame construction solutions in other systems. Finally, new tables are proposed with rankings of potential deficiencies for the CFS common building types consistent with existing tables for other building types. In Chapter 16, New Tier 1 checklists are proposed for the common building types. These checklists, which are dependent on desired performance level, provide potentially rapid screening of existing buildings.

Conclusions

The current version of ASCE 41: Seismic Evaluation and Retrofit of Existing Buildings (2013) provides limited guidance on the use of cold-formed steel and cold-formed steel framing. ASCE 41 is unique in that it specifically defines multiple performance levels that an engineer and owner may want to achieve, as a result it is being utilized in new design as well as in seismic retrofits. Significant additions have been proposed to the forthcoming (2017) addition of ASCE 41 for cold-formed steel. A large database (over 500 entries) of existing tests on cold-formed steel framed shear walls and strap-braced walls was gathered so that the full backbone response and related acceptance criteria could be developed for these systems in a manner consistent with ASCE 41's methodology. Related efforts on individual cold-formed steel members and steel-to-steel connections were also completed. In addition, common cold-formed steel building types, and the definition of damage and deterioration in the cold-formed steel components of these buildings were defined as needed for ASCE 41's evaluation procedures. Taken together the proposed efforts enable engineers to utilize or account for cold-formed steel in retrofit and new designs per the methodologies of ASCE 41.

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References

AISI S100 (2012) 'North American Specification for the Design of Cold-Formed Steel Structural Members', American Iron and Steel Institute, Washington, D.C.

AISI S213 (2007) 'North American Standard for Cold-Formed Steel Framing: Lateral Design', American Iron and Steel Institute, Washington, D.C.

AISI S400 (2015) 'North American Standard for Seismic Design of Cold-Formed Steel Structural Systems', American Iron and Steel Institute, Washington, D.C.

Al-Kharat, M., Rogers, C.A. (2005) 'Testing of light gauge steel strap braced walls', Project Report No. 1, McGill University, Montreal, Canada, August.

Al-Kharat, M., Rogers, C.A. (2006) 'Inelastic performance of screw connected light gauge steel strap braced walls', Project Report No. 2, McGill University, Montreal, Canada, December.

ASCE 7 (2002) 'Minimum Design Loads for Buildings and Other Structures' SEI/ASCE 7-02

ASCE 7 (2010) 'Minimum Design Loads for Buildings and Other Structures' SEI/ASCE 7-10

ASCE 41 (2013). 'Seismic Rehabilitation of Existing Buildings' ASCE/SEI 41-13

ASTM E2126 (2011) Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings, ASTM International.

ASTM E564 (2006) Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings, in ASTM E564, ASTM International: West Conshohocken, PA.

Ayhan, D., Schafer, B.W. (2016) 'Characterization of in-plane backbone response of cold-formed steel beams' (JCSR under review) [see CFS-NEES RR02 www.ce.jhu.edu/cfsnees/publications.php]

Balh, N., Rogers, C.A. (2010) 'Development of seismic design provisions for steel sheathed shear walls', Project Report, McGill University, Montreal, Canada, January.

Blais, C. (2006) 'Testing and analysis of light gauge steel frame / 9 mm OSB wood panel shear walls', Master Thesis, Advisor: Colin A. Rogers, McGill University, Montreal, Canada, January.

Boudreault, FA. (2005) ' Seismic analysis of steel frame / wood panel shear walls', Master Thesis, Advisor: Colin A. Rogers, McGill University, Montreal, Canada, June.

Branston, A.E. (2004) 'Development of a design methodology for steel frame / wood panel shear walls', Master Thesis, Advisor: Colin A. Rogers, McGill University, Montreal, Canada, June.

Chang, T.C. (2004) 'Testing and performance of steel frame / wod panel shear walls', Master Thesis, Advisor: Colin A. Rogers, McGill University, Montreal, Canada, August.

Comeau, G. (2008) 'Inelastic performance of welded cold-formed steel strap braced walls', Master Thesis, Advisor: Colin A. Rogers, McGill University, Montreal, Canada, June.

DaBreo, J., (2012) 'Impact of gravity loads on the lateral performance of cold-formed steel frame / steel sheathed shear walls', Master Thesis, Advisor: Colin A. Rogers, McGill University..
El-Saloussy, K. (2010) 'Additional cold-formed steel frame / steel sheathed shear walls design values for Canada', Master Thesis, Advisor: Colin A. Rogers, McGill University.
Elhaji, N. (2005) 'Cold-formed steel walls with fiberboard sheathing-shear wall testing', Summary Test Report, NAHB Research Center, Inc., Upper Malboro, Maryland, USA, September.
FEMA 302 (1997) 'NEHRP Recommended Provisions For Seismic Regulations For New Buildings And Other Structures', Federal Emergency Management Agency, Washington, D.C.
FEMA 356 (2007) 'Prestandard and Commentary for the seismic rehabilitation of buildings', Federal Emergency Management Agency, American Society of Civil Engineers (ASCE), Washington, D.C.
FEMA 450 (2003) 'NEHRP Recommended Provisions For Seismic Regulations For New Buildings And Other Structures', Federal Emergency Management Agency, Washington, D.C.
Harris, J.L., Speicher, M.S. (2015). "Assessment of First Generation Performance-Based Seismic Design Methods for New Steel Buildings, Vol. 1: Special Moment Frames." NIST TN 1863-1.
Hikita, K. (2006) 'Combined gravity and lateral loading of light gauge steel frame/wood panel shear walls', Master Thesis, Advisor: Colin A. Rogers, McGill University, Montreal, Canada, December.
IBC (2000) 'International Building Code', International Code Council, Inc., Falls Church, Virginia.
IBC (2003) 'International Building Code', International Code Council, Inc., Falls Church, Virginia.
Kochkin, V., Hill, R. (2006) 'Cyclic testing of fiberboard shear walls with varying aspect ratios', Report, NAHB Research Center, Inc., Upper Malboro, Maryland, USA, March.
Krawinkler, H., Parisi, F., Ibarra, L., Ayoub, A., and Medina, R., "Development of a Testing Protocol for Wood Frame Structures," CUREE Publication No. W-02, 2000.
Liu, P., Peterman, K.D., Schafer, B.W. (2012) 'Test report on cold-formed steel shear walls', Research Report, CFS-NEES project: NSF-CMMI-1041578.
Lu, S. (2015) 'Influence of gypsum panels on the response of cold-formed steel framed shear walls', Master Thesis, Advisor: Colin A. Rogers, McGill University, Montreal, Canada, May.
Moen, C.D., Tao, F., Colea, R. (2016). 'Monotonic and cyclic backbone response of single shear cold-formed steel screw-fastened connections', The International Colloquium On Stability and Ductility Of Steel Structures – SDSS 2016, 30 May- 01 June 2016, Timisoara, Romania.
Morello, D. (2009) 'Seismic performance of multi-story structures with cold-formed steel wood sheathed shear walls', Master Thesis, Advisor: Colin A. Rogers, McGill University.
Ong-Tone, C. (2009) 'Tests and evaluation of cold-formed steel frame/steel sheathed shear walls', Master Thesis, Advisor: Colin A. Rogers, McGill University, Montreal, Canada, April.
Rokas, D. (2006) 'Testing and evaluation of light gauge steel frame / 9.5 mm CSP wood panel shear walls', Master Thesis, Advisor: Colin A. Rogers, McGill University, Montreal, Canada, November.
Serrette, R., Enchalada, J., Hall, G., Matchen, B., Nyugen, H., Williams, A. (1997) 'Additional shear wall values for light weight steel framing', Report No. LGSRG-I-97, Department of Civil Engineering, Santa Clara University, Santa Clara, California, USA, March.
Shamim, I. (2012) 'Seismic design of lateral force resisting cold-formed steel framed structures', Doctor of Philosophy Thesis, Advisor: Colin A. Rogers, McGill University.
UBC (1997) 'Uniform Building Code', International Conference of Building Officials (ICBO), Whittier, California.
Velchev, K. (2008) 'Inelastic performance of screw connected cold-formed steel strap braced walls', Master Thesis, Advisor: Colin A. Rogers, McGill University, Montreal, Canada, November.
Yu, C., Chen, Y. (2009) 'Steel sheathing options for cold-formed steel framed shear walls assemblies providing shear resistance - Phase 2', Report No. UNT-G70752, University of North Texas.
Yu, C., Vora, H., Dainard, T., Tucker, J., Veetvkuri, P. (2007) 'Steel sheathing options for cold-formed steel framed shear walls assemblies providing shear resistance', Report No. UNT-G76234, Department of Engineering Technology, University of North Texas, Denton, Texas, USA, October.
Zhao, Y., Rogers, C. A. (2002) 'Cyclic performance of cold-formed steel stud shear walls', Progress Report No. 2, McGill University, Montreal, Canada, September.